# Contents



*3C:P3:HRM3*

## 3-1 Introduction

The purpose of this chapter is to define the computational standards required for the design of stormwater treatment and detention facilities. Within the chapter is an explanation of the method that is to be used for the design of stormwater facilities and the supporting data that will be needed to complete the design.

The method of analysis presented in this chapter is the Santa Barbara Urban Hydrograph (SBUH) method. It currently represents the best workable process for calculating total runoff during a storm event for a small basin. Designing a stormwater runoff facility using the volume of a storm event leads to significantly better results than designing the facility based on a peak intensity. Since the SBUH method is an improvement over methods based on the Rational Formula, it supersedes the method for stormwater facility design presented in Chapter 7 of the Washington State Department of Transportation (WSDOT) *Hydraulics Manual*.

The SBUH method is very computationally intensive. Calculations for even a single area would take several hours if done with a standard calculator. As a result of this, the only practical way to conduct an analysis is to use a computer application. The equations used are such that they could be incorporated into a spreadsheet program to provide the necessary computations. However, it is highly recommended that one of the commercially available computer programs that offers the SBUH method for analysis be used. The advantage of using commercial software is the overall consistency of input and output formats and the reliability of being tested in several different design circumstances. The Olympia Service Center Hydraulics Section uses, and encourages designers in the regional offices to use, the software package WaterWorks written by Engenious Systems. The examples presented in this chapter assume that the designer is using WaterWorks.

## 3-2 Project Considerations

Prior to conducting any stormwater runoff calculations, the overall relationship between the proposed project and the runoff that it will be creating must be considered. When the project layout is first being determined, estimates of the area that will be required for stormwater treatment must be known in order to provide for adequate purchase of right of way for the project. To successfully estimate the required area, several items need to be covered. The basic requirements for the stormwater facility design must be known, the general hydrologic characteristics of the site where the project is located must be known, and the basic alignment of the new project must be known.

In most instances, the basic requirements for stormwater facilities described within this manual will be adequate to meet other state agency and local jurisdiction requirements. Some projects will be located in areas that have been designated as requiring more stringent runoff control standards. The first part of any hydrologic analysis will be conducting research to determine if the project is located in an area where additional requirements exist. Typically, this can be accomplished by consulting with the district hydraulics or environmental sections.

When stricter standards do apply, they are usually in the form of increased rainfall duration or lower site discharge rates. Either case is easily applied to the method of analysis outlined in this chapter.

The basic hydrologic characteristics of the project site will dictate the amount of runoff that will occur and where stormwater facilities can be placed. Several sources exist that will be useful in determining the necessary information for runoff analysis. Drainage patterns and contributing areas can be determined from contour maps that were generated from preliminary surveys of the area for the proposed project or from contour maps for a previous project in the same area. Soil characteristics can be found in Soil Conservation Service (SCS) publications or from analysis done by the Materials Lab. Existing drainage facilities and conveyance system locations can be found in Hydraulics Reports from previous projects in the same vicinity or in plans for the existing roadway. Another source of information is the outfall inventory/field screening database. The final part of determining site characteristics is to visit the site of the proposed project. The field visit will serve to verify all of the information that was obtained through research and will show where that information may have been deficient. In nearly every instance, the information gained by visiting the site prior to designing the stormwater facilities is far more beneficial than the calculations that could have been performed by the designer if he/she had remained in the office.

Once the basic stormwater requirements are understood and the general hydrologic site characteristics are known, the necessary area for stormwater facilities can be estimated. This is done by examining the proposed layout of the project and determining the most suitable locations to place the facilities. Then the method described later in this chapter can be applied to the site and an estimate of the required area can be calculated. If this process is done early enough, then slight alterations to the project alignment can be made and adequate right of way can be purchased. A final design of the stormwater facilities will have to be performed when the project layout is finalized. The location of stormwater outfalls should be provided to local agencies and added to WSDOT's outfall inventory to facilitate compliance with NPDES and Highway Runoff Rule requirements.

## 3-3 Hydrograph Method

To correctly size an inlet, a pipe, or a ditch for conveyance of highway drainage, the designer needs to know the peak flow that will be required to pass through the system. Since only a single flow value is needed, a method such as the intensity duration based Rational Method presented in Chapter 2 of the WSDOT *Hydraulics Manual* will yield correct sizes. To correctly size some of the BMPs presented later in this manual, the designer must know the peak flow that will be generated from the site along with the total volume of runoff that will be generated and the timing that different flow rates will occur during the storm event. While there are ways to synthesize the required information from the results of a Rational Method analysis, it is more accurate to use a method that generates a hydrograph, the standard plot of runoff flow versus time, based on a volume of rainfall for a given time period.

At this time, the SBUH method represents the best approach for designing highway runoff BMPs which require a hydrograph analysis. The SBUH method models runoff by analyzing a given time period of rainfall to generate a hydrograph which is sensitive to variations in the rainfall preceding and following the peak unlike intensity duration models which are only sensitive to the peak rainfall intensity. It was specifically developed to model runoff from an urbanized, mostly impervious land use, unlike other popular event based models which where developed to model runoff from agricultural land uses. The SBUH method has easily understood input parameters unlike the data intensive and extremely complicated continuous simulation models.

A SBUH analysis requires that the designer understands certain characteristics of the project site such as drainage patterns, predicted rainfall, soil type, area to be covered with impervious material, method of drainage conveyance, and the BMP that will be used. The physical characteristics of the site and the design storm determine the magnitude, volume, and duration of the runoff hydrograph. Other factors such as the conveyance characteristics of channel or pipe, merging tributary flows, and type of BMP used will alter the shape and magnitude of the hydrograph. In the following sections, the key elements of hydrograph analysis are presented, namely:

- Design storm hyetograph
- Runoff parameters
- Hydrograph synthesis
- Hydrograph routing
- Hydrograph summation

### 3-3.1 Design Storm Hyetograph

The SBUH method requires the input of a rainfall distribution or design storm hyetograph. The design storm hyetograph is rainfall depth versus time for a given design storm frequency and duration. For this application, it is presented as a dimensionless table of unit rainfall depth (increment rainfall depth for each time interval divided by the total rainfall depth) versus time.

The SCS has developed several hyetographs that are used throughout the county for runoff calculations. For projects in western Washington, the SCS Type 1A hyetograph should be used. Eastern Washington projects should use the SCS Type 2 hyetograph. The reason for using different hyetographs is to account for the different types of rainfall patterns that occur throughout the state. Both of these hyetographs have a duration of 24 hours and both are encoded into the WaterWorks computer program.

The design storm hyetograph is constructed by multiplying the dimensionless hyetograph by the rainfall depth (in inches). The total depth of rainfall for storms of 24-hour duration and 2-, 10-, 25-, and 100-year recurrence intervals are presented in Figure 3-3.1. Obtain the value from the figure for the city that is closest to the project site. Another method for obtaining rainfall volumes for different storms is to use Isopluvial maps. Isopluvial maps for different storm durations and recurrence intervals are published by the National Weather Service. This information can be obtained from the Olympia Service Center Hydraulics Section.

| Location | 2-year | 10-year | 25-year | 100-year |
|---|---|---|---|---|
| Aberdeen and Hoquiam | 2.5 | 3.2 | 4.0 | 5.0 |
| Anacortes | 1.2 | 2.0 | 2.5 | 3.1 |
| Bellingham | 1.8 | 2.7 | 3.1 | 3.8 |
| Bremerton | 2.5 | 3.5 | 4.5 | 5.0 |
| Cathlamet | 3.5 | 5.0 | 5.7 | 6.8 |
| Centralia and Chehalis | 2.2 | 3.0 | 3.5 | 4.3 |
| Clarston and Colfax | 1.4 | 1.8 | 2.1 | 2.6 |
| Colville | 1.4 | 1.9 | 2.2 | 2.6 |
| Ellensburg | 1.5 | 2.3 | 2.7 | 3.5 |
| Elma | 3.0 | 4.2 | 4.7 | 6.0 |
| Everett | 1.5 | 2.3 | 2.6 | 3.2 |
| Forks | 5.5 | 7.5 | 8.5 | 10.0 |
| Gold Bar | 3.0 | 3.5 | 4.0 | 5.0 |
| Goldendale | 1.5 | 2.3 | 2.8 | 3.5 |
| Hoffstadt Creek | 4.5 | 6.0 | 7.5 | 9.0 |
| Hoodsport | 4.8 | 6.6 | 7.7 | 9.5 |
| Humptulips | 5.0 | 6.7 | 8.0 | 10.0 |
| Kelso and Longview | 2.5 | 3.5 | 4.0 | 5.0 |
| Leavenworth | 1.5 | 2.0 | 2.5 | 3.0 |
| Long Beach | 3.0 | 4.0 | 4.5 | 5.5 |
| Moses Lake | 0.7 | 1.1 | 1.3 | 1.7 |
| Mount Vernon | 1.7 | 2.6 | 3.0 | 3.7 |
| Naselle | 4.5 | 6.0 | 7.0 | 8.5 |
| North Bend | 4.0 | 5.0 | 6.0 | 6.5 |
| Olympia | 2.8 | 4.3 | 5.1 | 6.1 |
| Omak | 1.0 | 1.6 | 1.8 | 2.2 |
| Pasco and Richland | 0.8 | 1.3 | 1.6 | 2.0 |
| Port Angeles | 2.1 | 3.1 | 3.9 | 4.6 |
| Port Townsend | 1.0 | 1.7 | 2.0 | 2.5 |
| Poulsbo | 2.0 | 2.4 | 3.5 | 4.0 |
| Queets | 5.2 | 6.5 | 7.5 | 9.3 |
| Raymond | 3.5 | 4.9 | 5.5 | 6.5 |
| Seattle | 2.0 | 2.8 | 3.4 | 4.0 |
| Sequim | 1.5 | 2.0 | 2.5 | 2.8 |
| Shelton | 4.0 | 5.5 | 6.5 | 8.5 |
| Snoqualmie Pass | 5.5 | 7.0 | 8.0 | 10.0 |
| Spokane | 1.4 | 2.0 | 2.2 | 2.6 |
| Stevens Pass | 5.0 | 6.5 | 7.5 | 9.0 |
| Sumas | 2.5 | 3.3 | 3.8 | 4.5 |
| Sumner | 2.0 | 2.8 | 3.3 | 3.9 |
| Tacoma | 2.0 | 3.0 | 3.5 | 4.0 |
| Toledo | 2.3 | 3.0 | 3.5 | 4.2 |
| Vancouver | 2.3 | 3.0 | 3.5 | 4.3 |
| Vantage | 1.0 | 1.6 | 1.9 | 2.3 |
| Walla Walla | 1.2 | 1.9 | 2.0 | 2.4 |
| Wenatchee | 1.5 | 2.2 | 2.5 | 3.1 |
| White Pass | 4.5 | 6.0 | 7.0 | 8.0 |
| Yakima | 1.0 | 1.5 | 1.8 | 2.2 |

**24-Hour Duration Rainfall Volumes**

***Figure 3-3.1***

***Example***

Determine the 100-year design storm depth for a project:

1. Located Near Tacoma

   100-year 24-hour rainfall depth = 4.0 inches (102 mm)

2. Located Near Spokane

   100-year 24-hour rainfall depth = 2.6 inches (66 mm)

When a project requires a runoff quantity control BMP as described in Minimum Requirement 5 in Chapter 2, an SBUH method design must be done using the 2-year, 10-year, and 100-year return frequencies. The rainfall depths for the different storm recurrence intervals are obtained as previously explained. To design a water quality control BMP as described in Minimum Requirement 4 in Chapter 2, an SBUH method design may be required. If the BMP selected for use requires a hydrograph method design, then the correct hyetograph to use is the 6-month recurrence storm. The 6-month rainfall depth is equal to 64 percent of the 2-year, 24-hour volume.

***Example***

Determine the 6-month design storm depth for a project near Tacoma:

2-year, 24-hour rainfall depth = 2.0 inches

6-month design storm = (.64)(2yr 24hr storm)
= (.64)(2.0)
= 1.3 inches (33 mm)

For project sites with tributary drainage areas above elevation 1,000 feet Mean Sea Level, the designer must consider the effects of snow melt. At higher elevations, the worst cases of runoff often occur when warm rain from the design storm falls on snow, causing runoff not only from the precipitation of the storm but also from the melting snow. An inch of rain is roughly equivalent to 10 inches of snow. By increasing the 24-hour rainfall depth 1 to 2 inches, the effects of snow melt are usually well accounted for. There should be no increase applied to roadway sections that will have snow removed during the winter months.

## 3-3.2 Runoff Parameters

The SBUH method requires input of parameters which describe physical drainage basin characteristics. These parameters provide the basis from which the runoff hydrograph is developed. This section describes the three key parameters (area, curve number, and time of concentration) that along with the rainfall hyetograph develop the runoff hydrograph through use of the SBUH method.

The proper selection of contributing basin areas is required to obtain a high degree of accuracy in hydrograph analysis. The basin area used should be relatively homogeneous in land use and soil type. If the entire contributing basin is similar in these aspects, then the basin can be analyzed as a single area. If significant differences exist within a given drainage basin, then it must be divided into subbasin areas of similar runoff characteristics. For example, a drainage basin, consisting of a park and ride lot and a large forested area should be divided into two subbasin areas accordingly. Hydrographs should then be computed for each subbasin area and summed to form the total runoff hydrograph for the basin. Drainage basins larger than 10 acres should be divided into subbasins. By dividing large basins into smaller subbasins and then combining calculated flows, the timing aspect of the generated hydrograph is typically more accurate.

Basin areas can be determined with a contour map. Contour maps that are generated specifically for the project site are the most accurate way of obtaining the drainage area since they are done with 5-foot or less contours. If the drainage area extends past the limits of the maps generated for the project, then USGS Quadrangle contour maps can be used to obtain the basin area. New impervious area should always be measured from project specific maps.

To determine the basin area contributing to a project, the area first must be outlined on the contour map. This is done by locating the project's discharge point on the map then drawing a line along the ridge line of the basin, finally connecting back to the discharge point. This will need to be done for each discharge point of the project site. If the flow from two or more discharge points can be combined, then their basins can also be combined. Once the basin boundary is drawn on a map, it can be measured using a planimeter or digitized on a CAD station and scaled.

The SCS has conducted studies into the runoff characteristics of various land types. After gathering and analyzing extensive data, the SCS developed relationships between land use, soil type, vegetation cover, interception, infiltration, surface storage, and runoff. The relationships have been characterized by a single runoff coefficient called a curve number (CN). Figure 3-3.2 shows suggested CN values for various land covers.

The factors that contribute to the CN value are known as the soil-cover complex. The soil-cover complexes have been assigned to one of four hydrologic soil groups, according to their runoff characteristics. These soil groups are labeled Type A, B, C, and D with Type A generating the least amount of runoff and Type D generating the greatest. Figure 3-3.3 shows the hydrologic soil group of most soils in Washington State. The different soil groups can be described as:

- Type A — Soils having high infiltration rates, even when thoroughly wetted, and consisting chiefly of deep, well-to-excessively drained sands or gravels. These soils have a high rate of water transmission.
- Type B — Soils having moderate infiltration rates when thoroughly wetted, and consisting chiefly of moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
- Type C — Soils having slow infiltration rates when thoroughly wetted, and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine textures. These soils have a slow rate of water transmission.
- Type D — Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a hardpan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

| Land Use Description | | Curve Numbers by Hydrologic Soil Group | | | |
|---|---|---|---|---|---|
| | | **A** | **B** | **C** | **D** |
| Mountain brush — oak brush, aspen, maple: | | | | | |
| Good Condition: | ground cover > 70% | 40 | 40 | 41 | 48 |
| Fair Condition: | ground cover 30% to 70% | 40 | 48 | 57 | 63 |
| Poor Condition: | ground cover < 30% | 46 | 66 | 74 | 79 |
| Woods or forest land: | | | | | |
| Good condition: | Natural conditions | 40 | 55 | 70 | 77 |
| Fair condition: | Some forest litter | 40 | 60 | 73 | 79 |
| Poor condition: | No small trees or brush | 45 | 66 | 77 | 83 |
| Woods/Grass combination (orchard or tree farm): | | | | | |
| Good condition: | ground cover > 75% | 40 | 58 | 72 | 79 |
| Fair condition: | ground cover 50% to 75% | 43 | 65 | 76 | 82 |
| Poor condition: | ground cover < 50% | 57 | 73 | 82 | 86 |
| Brush with weeds and grass: | | | | | |
| Good condition: | ground cover > 75% | 40 | 48 | 65 | 73 |
| Fair condition: | ground cover 50% to 75% | 40 | 56 | 70 | 77 |
| Poor condition: | ground cover < 50% | 48 | 67 | 77 | 83 |
| Meadow — continuous grass: | | 40 | 58 | 71 | 78 |
| Residential districts: | | | | | |
| ¼ acre lots: | | 61 | 75 | 83 | 87 |
| ⅓ acre lots: | | 57 | 72 | 81 | 86 |
| ½ acre lots: | | 54 | 70 | 80 | 85 |
| 1 acre lots: | | 51 | 68 | 79 | 84 |
| Pasture or range: | | | | | |
| Good condition: | lightly grazed | 40 | 61 | 74 | 80 |
| Fair condition: | not heavily grazed | 49 | 69 | 79 | 84 |
| Poor condition: | heavily grazed w/no mulch | 68 | 79 | 86 | 89 |
| Newly graded areas (no vegetation established) | | 77 | 86 | 91 | 94 |
| Open spaces, lawns, parks, golf courses, cemeteries: | | | | | |
| Good condition: | grass cover > 75% | 40 | 61 | 74 | 80 |
| Fair condition: | grass cover 50% to 75% | 49 | 69 | 79 | 84 |
| Poor condition: | grass cover < 50% | 68 | 79 | 86 | 89 |
| Gravel roads and parking lots: | | 88 | 92 | 95 | 98 |
| Dirt roads and parking lots: | | 86 | 90 | 94 | 98 |
| Impervious surfaces: | pavement and roofs | 98 | 98 | 98 | 98 |
| Open water bodies: | lakes, wetlands, and ponds | 100 | 100 | 100 | 100 |

For a more detailed description of agricultural land use and arid region curve numbers, refer to Soil Conservation Service Technical Release 55, Chapter 2, June 1986.

**Runoff Curve Numbers**

***Figure 3-3.2***

| Soil Name | Class | Soil Name | Class | Soil Name | Class |
|---|---|---|---|---|---|
| Aabab | B | Bograp | B | Clayton | B |
| Adkins | B | Boistfort | C | Cle Elum | B |
| Aeneas | B | Bong | B | Cleman | B |
| Agnew | C | Bonner | B | Clint | B |
| Ahl | B | Bossburg | D | Cloquato | B |
| Ahren | B | Bow | C | Cocolalla | C |
| Ahtanum | C | Brew | C | Colockum | B |
| Aita | C | Brickel | C | Colter | B |
| Ats | B | Bridgeson | C | Colville | D |
| Alderwood | C | Brief | B | Conconully | B |
| Almota | C | Briscot | C | Copalis | B |
| Alpowa | B | Broadax | C | Cordy | B |
| Alvor | C | Bromo | B | Cotteral | B |
| Ampad | C | Buckhorn | B | Covello | B |
| Anatone | D | Buckley | C | Cowinch | B |
| Anders | C | Buhrig | C | Cusick | D |
| Ansaldo | B | Bunker | B | Custer | C |
| Antilon | B | Burbank | A | Dabob | C |
| Ardenvoir | B | Burch | B | Dalkena | B |
| Arents | B | Burke | C | Darland | B |
| Arta | C | Cagey | C | Dart | A |
| Ashoe | B | Calawah | B | Dearyton | B |
| Asotin | C | Calcuse | B | Dehart | B |
| Babcock | C | Carlsborg | A | Delphi | D |
| Badge | B | Carmack | B | Dick | A |
| Bagdad | B | Carstairs | A | Dimal | D |
| Baldhill | B | Casey | C | Dinkelman | B |
| Bamber | C | Cashmere | B | Dinkels | B |
| Barneston | A | Casmont | B | Disautel | B |
| Baumgard | B | Cassolary | B | Dobbs | B |
| Bear Prairie | B | Cathcart | C | Dome | B |
| Beausite | B | Cathlamet | B | Donavan | B |
| Beckley | B | Catla | D | Doty | B |
| Belfast | B | Cattcreek | B | Dougan | B |
| Beljica | B | Cedonia | B | Dougville | B |
| Bellicum | B | Centralia | B | Dragoon | B |
| Bellingham | C | Chamokane | B | Dungeness | B |
| Belzar | C | Chard | B | Dupont | D |
| Benco | B | Chehalis | B | Earlmont | C |
| Benge | B | Chelan | B | Edgewick | C |
| Benham | B | Chesaw | A | Ekrue | C |
| Bernhill | B | Chewawa | B | Eld | B |
| Bestrom | B | Cheney | B | Ellisforde | C |
| Beverly | B | Chesaw | A | Elochoman | B |
| Bickleton | B | Chewelah | B | Eloika | B |
| Bisbee | A | Chiwawa | B | Elwell | C |
| Bjork | C | Cinebar | B | Elwha | C |
| Boistfort | B | Cispus | A | Emdent | C |
| Booker | D | Clallam | C | Endicott | C |
| Boesel | B | Clato | B | Entiat | D |

**Hydrologic Soil Groups**

***Figure 3-3.3***

| Soil Name | Class | Soil Name | Class | Soil Name | Class |
|---|---|---|---|---|---|
| Ephrata | B | Hoypus | A | Ledow | A |
| Esquatzel | B | Huckleberry | C | Lickskillet | D |
| Everett | A | Huel | A | Linvelle | B |
| Everson | C | Hum | B | Littlejohn | B |
| Ewall | A | Humptulips | B | Logy | B |
| Farrell | B | Hunters | B | Loneridge | C |
| Finely | B | Hyas | B | Louella | B |
| Foss | B | Ilwaco | B | Lummi | D |
| Galvin | C | Indianola | A | Lynnwood | A |
| Garfield | C | Inkler | B | Lyre | A |
| Garrison | B | Jimcom | C | Lystair | B |
| Germany | B | Jonas | B | Lytell | B |
| Getchell | C | Jumpe | B | Mabton | B |
| Giles | B | Juno | A | Magallon | B |
| Glenoma | B | Kalaloch | B | Makah | B |
| Godfrey | D | Kapowsin | C | Maki | C |
| Gorskel | C | Karamin | B | Mal | B |
| Gorst | D | Kartar | B | Malaga | B |
| Govan | C | Kartar | B | Manley | B |
| Green Bluff | B | Katula | B | Marble | A |
| Greenwater | A | Kegel | C | Margerum | B |
| Grehalem | B | Kennewick | B | Marlin | D |
| Grisdale | B | Kiehl | A | Martella | B |
| Grove | A | Kilchis | C | Mashel | B |
| Gwin | D | Kiona | B | Maytown | C |
| Hagen | B | Kitsap | C | McDaniel | B |
| Hakker | C | Kittitas | D | McElroy | B |
| Halbert | D | Klaber | D | McKenna | C |
| Hale | B | Klaus | B | McMurray | D |
| Haley | B | Klicker | B | Melbourne | B |
| Hanning | B | Klone | B | Menzel | B |
| Hardesty | B | Knappton | B | Merkel | B |
| Harstine | C | Koehler | C | Meystre | C |
| Hartill | B | Koepke | B | Mikkald | C |
| Hartline | B | Koerling | B | Mippon | B |
| Hartnit | C | Konert | C | Mires | B |
| Harwood | C | Kohner | D | Mobate | C |
| Hatchey | C | Koseth | B | Molcal | B |
| Havillah | B | Kuhl | D | Molson | B |
| Hesseltine | B | Kydaka | D | Mondovi | B |
| Hesson | C | Lacamas | D | Montesa | C |
| Heytou | B | Laketon | B | Mopang | B |
| Hezel | B | Lance | B | Morical | C |
| Hodgson | C | Latah | C | Moscow | B |
| Hoffstadt | B | Lates | B | Mossyrock | B |
| Hoh | B | Le Bar | B | Mowich | C |
| Hoko | C | Leader | B | Moxee | D |
| Hoodsport | C | Leadpoint | B | Mukilteo | D |
| Hoogdal | C | Leavenworth | B | Murnen | B |
| Hoquiam | B | Lebam | B | Naches | B |

**Hydrologic Soil Groups**

***Figure 3-3.3***

| Soil Name | Class | Soil Name | Class | Soil Name | Class |
|---|---|---|---|---|---|
| Naff | B | Papac | B | Sagehill | B |
| Narcisse | B | Para | B | Sagemore | B |
| Nard | B | Pastik | C | Salal | C |
| Narel | B | Patit Creek | B | Salkum | C |
| Nargar | A | Pedigo | C | Saltese | D |
| National | B | Peoh | C | Salzer | D |
| Naxing | B | Peone | C | Sammamish | D |
| Neilton | A | Pheeney | B | San Juan | A |
| Nemah | C | Phelan | D | Sarkin | C |
| Neppel | B | Phoebe | B | Satsop | B |
| Nesika | B | Pickett | C | Sauvola | B |
| Nespeleh | B | Pilchuck | A | Saydab | C |
| Netrac | A | Pitcher | B | Scamman | C |
| Nevat | A | Pogue | B | Schawana | C |
| Nevine | B | Potchub | C | Schneider | B |
| Newaukum | B | Poulsbo | C | Schnorbush | B |
| Newberg | B | Prather | C | Schooley | D |
| Newbon | B | Prosser | C | Schumacher | B |
| Newruss | B | Prouty | B | Scoap | B |
| Newskah | B | Puget | C | Scoon | D |
| Nighthawk | B | Puyallup | B | Scocteney | B |
| Nimue | B | Queets | B | Scotia | B |
| Nisqually | A | Quilcene | C | Scrabblers | B |
| Nooksack | C | Quillayute | B | Sealak | D |
| Nordby | A | Quincy | A | Seastrand | C |
| Norma | B | Ragnar | A | Seattle | D |
| Novark | B | Rainer | C | Sekiu | D |
| Nuby | C | Raisic | B | Selah | D |
| O'Brien | B | Ralls | B | Semiahmoo | D |
| Ocosta | C | Raught | B | Sequim | A |
| Odo | B | Reardan | C | Shalcar | D |
| Ogarty | C | Reed | D | Shano | B |
| Ohana | C | Reichel | B | Shelton | C |
| Okanogan | B | Rennie | D | Si | C |
| Olequa | B | Renslow | B | Sifton | B |
| Olete | C | Renton | D | Siler | B |
| Olomount | C | Republic | B | Simode | C |
| Olympic | B | Ret | C | Sinclair | C |
| Onyx | B | Risbeck | B | Sinloc | B |
| Orcas | D | Ritel | C | Skaha | A |
| Oridia | D | Ritzville | B | Skamania | B |
| Orting | C | Rober | C | Skamo | B |
| Oso | C | Rock Creek | D | Skanid | C |
| Outlook | C | Roloff | C | Skate | B |
| Ovall | C | Royal | B | Skeller | B |
| Owhi | B | Roza | D | Skipopa | D |
| Oyhut | B | Rufus | C | Skoly | B |
| Ozette | C | Rugles | B | Skykomish | B |
| Palix | B | Sacheen | A | Smackout | B |
| Palouse | B | Sadie | C | Snahopish | B |

**Hydrologic Soil Groups**

***Figure 3-3.3***

| Soil Name | Class | Soil Name | Class | Soil Name | Class |
|---|---|---|---|---|---|
| Snohomish | D | Tebo | B | Waha | C |
| Snow | B | Tekison | C | Wahkiakum | B |
| Solduc | A | Tekoa | C | Wahlike | C |
| Solleks | B | Tenino | C | Wahtum | D |
| Spana | B | Terbies | B | Waits | B |
| Spanaway | A | Thout | C | Walla Walla | B |
| Speigle | B | Thow | B | Walville | B |
| Spens | B | Thrash | B | Wamea | B |
| Spofford | C | Tieton | B | Wanser | B |
| Spokane | B | Timberhead | B | Wapato | D |
| Springdale | B | Timentwa | B | Warden | B |
| Spukwush | B | Timmerman | A | Washougal | B |
| Squally | B | Tisch | C | Weirman | B |
| St. Martin | C | Tokul | C | Wellman | A |
| Stabler | B | Tonasket | B | Wenas | C |
| Stahl | C | Toppenish | B | Westport | B |
| Staley | B | Touhey | B | Wethey | B |
| Standup | B | Townsend | C | Whidbey | C |
| Starbuck | D | Track | B | White Swan | C |
| Startup | B | Tradedollar | B | Wiehl | C |
| Steever | B | Traham | B | Wilkeson | C |
| Stemilt | C | Triton | D | Willaby | B |
| Stevens | B | Tucannon | C | Willapa | C |
| Stevenson | B | Tukwila | D | Willis | C |
| Stimson | C | Tukey | C | Winchester | A |
| Strat | B | Tumac | B | Winston | A |
| Stratford | B | Tyee | D | Winthrop | A |
| Sulsavar | B | Uhlig | B | Wintoner | C |
| Sultan | C | Umapine | C | Wishkah | D |
| Sumas | C | Underwood | B | Wolfeson | C |
| Supplee | B | Undusk | B | Woodinville | B |
| Sutkin | B | Vader | B | Yakima | B |
| Swantown | D | Vailton | B | Yaxon | B |
| Swem | C | Vallan | D | Yeary | C |
| Sylvia | C | VanZandt | B | Yelm | C |
| Synarep | B | Varelum | C | Yost | C |
| Tacoma | D | Vassar | B | Zen | C |
| Taneum | C | Verlot | C | Zenker | B |
| Tanwax | D | Vesta | B | Zillah | B |
| Taunton | C | Voight | B | Zynbar | B |
| Tealwhit | C | Wadams | B | Zyzyl | B |

**Hydrologic Soil Groups**

***Figure 3-3.3***

The SCS has developed maps for Washington State that show the specific soil classification for any given location. These maps are compiled by county and are typically available from the regional SCS office. To determine which soil group to use for an analysis, locate the project site on the SCS map and read the soil classification that is listed. Use Figure 3-3.3 to convert from the specific soil classification to a hydrologic soil group. The WSDOT Materials Lab can also perform a soil analysis to determine the soil group for the project site. This should only be done if an SCS soils map cannot be located for the county which the site is in or if there is a reason to doubt the accuracy of the information on the SCS map for the particular site.

When performing an SBUH analysis for a basin, it is common to encounter more than one soil type within that basin. If the soil types are fairly similar (within 20 CN points), then a weighted average can be used. If the soil types are significantly different, then the basin should be separated into smaller subbasins, just as previously described for different land uses. Pervious and impervious soil types should always be analyzed separately. If the computer program WaterWorks is used for the analysis, then pervious and impervious land segments will automatically be separated but similar pervious soil types for a basin will have to be combined and weighted manually by the designer.

***Example***

Select CN values for a project near Tacoma:

Existing Land Use — forest (fair condition)

Basin Size — 10 acres

Future Impervious — 3.9 acres

Soil Type — 80% Alderwood, 20% Melbourne

Figure 3-3.3 shows that Alderwood soil belongs to the “C” hydrologic soil group and Melbourne soil belongs to the “B” group. Therefore, for the existing condition, CNs of 73 and 60 are read from Figure 3-3.2 and weighted to obtain a CN value of 70. For the developed condition, the impervious pavement for the interchange totals 3.9 acres. The 6.1 acres of pervious area consists of grassed open space in fair condition covering the same proportions of Alderwood and Melbourne soil. Therefore, CNs of 79 and 69 are read from Figure 3-3.2 and weighted to obtain a pervious area CN value of 77. The impervious area CN value is 98. The result of this example is summarized below:

| On-Site Condition | Existing | Developed |
|---|---|---|
| Land use | Forest | Interchange |
| Pervious area | 10 ac. | 6.1 ac. |
| CN of pervious area | 70 | 77 |
| Impervious area | 0 ac. | 3.9 ac. |
| CN of impervious area | — | 98 |

Travel time ($T_t$) is the time it takes water to travel from one location to another in a watershed. $T_t$ is a component of time of concentration ($T_c$), which is the time it takes for runoff to travel from the hydraulically most distant point of the watershed. $T_c$ is computed by summing all the travel times for consecutive components of the drainage conveyance system. $T_c$ influences the shape and peak of the runoff hydrograph. Urbanization usually decreases $T_c$, thereby increasing peak discharge.

Water moves through a watershed as sheet flow, shallow concentrated flow, open channel flow, or some combination of these. The type of flow that occurs is best determined by field inspection.

Sheet flow is flow over plane surfaces. It usually occurs in the headwater of streams. With sheet flow, the friction value ($n_s$) (a modified Manning's roughness coefficient) is used. These $n_s$ values are for very shallow flow depths of about 0.1 foot (3 cm) and are only used for travel lengths up to 300 feet (90 m). Figure 3-3.4 gives Manning's $n_s$ values for sheet flow for various surface conditions.

For sheet flow of up to 300 feet, use Manning's kinematic solution to directly compute $T_t$.

$$T_t = (0.42\ (n_s L)^{0.8})/((P_2)^{0.527}(s_o)^{0.4})$$

where:

$T_t$ = travel time (min)
$n_s$ = sheet flow Manning's coefficient
$L$ = flow length (ft)
$P_2$ = 2-year, 24-hour rainfall (in)
$s_o$ = slope of hydraulic grade line (land slope, ft/ft)

After a maximum of 300 feet, sheet flow is assumed to become shallow concentrated flow. The average velocity for this flow can be calculated using the $k_s$ values from Figure 3-3.4 in which average velocity is a function of watercourse slope and type of channel. After computing the average velocity using the Velocity Equation, the travel time ($T_t$) for the shallow concentrated flow segment can be computed by dividing the length of the segment by the average velocity.

Open channels are assumed to begin where surveyed cross section information has been obtained, where channels are visible on aerial photographs, or where lines indicating streams appear on USGS Quadrangle maps. The $k_c$ values from Figure 3-3.4 used in the Velocity Equation can be used to estimate average flow velocity. Average flow velocity is usually determined for bankfull conditions. After average velocity is computed, the travel time ($T_t$) for the channel segment can be computed by dividing the length of the channel segment by the average velocity.

A commonly used method of computing average velocity of flow, once it has measurable depth, is the Velocity Equation:

$$V = (k)(s_o^{0.5})$$

where:

$V$ = velocity (ft/s)
$k$ = time of concentration velocity factor (ft/s)
$s_o$ = slope of flow path (ft/ft)

The following limitations apply in estimating travel time ($T_t$).

- Manning's kinematic solution should not be used for sheet flow longer than 300 feet.
- The equations given here to calculate velocity where developed by empirical means, as a result the values that are put in must be in English Units (such as inches) for the equation to yield a correct answer. Once the velocity is calculated, it can be converted to metric units to finish the travel time calculations in the case of shallow concentrated flow and channel flow.

| **Sheet Flow Equation Manning's** | $\mathbf{n_s}$ |
|---|---|
| Smooth surfaces (concrete, asphalt, gravel) | 0.011 |
| Fallow fields or loose soil surface (no residue) | 0.05 |
| Cultivated soil with residue cover (s< 0.20 ft/ft) | 0.06 |
| Cultivated soil with residue cover (s> 0.20 ft/ft) | 0.17 |
| Short prairie grass and lawns | 0.15 |
| Dense grasses | 0.24 |
| Bermuda grass | 0.41 |
| Range (natural) | 0.13 |
| Woods or forest with light underbrush | 0.40 |
| Woods or forest with dense underbrush | 0.80 |
| **Shallow Concentrated Flow** | $\mathbf{k_s}$ |
| 1. Forest with heavy ground litter and meadows (n = 0.10) | 3 |
| 2. Brushy ground with some trees (n = 0.060) | 5 |
| 3. Fallow or minimum tillage cultivation (n = 0.040) | 8 |
| 4. High grass (n = 0.035) | 9 |
| 5. Short grass, pasture and lawns (n = 0.030) | 11 |
| 6. Nearly bare ground (n = 0.25) | 13 |
| 7. Paved and gravel areas (n = 0.012) | 27 |
| **Channel Flow (Intermittent)** | $\mathbf{k_c}$ |
| 1. Forested swale with heavy ground litter (n = 0.10) | 5 |
| 2. Forested drainage course with defined channel bed (n = 0.050) | 10 |
| 3. Rock-lined waterway (n = 0.035) | 15 |
| 4. Grassed waterway (n = 0.030) | 17 |
| 5. Earth-lined waterway (n = 0.025) | 20 |
| 6. CMP pipe (n = 0.024) | 21 |
| 7. Concrete pipe (0.012) | 42 |
| **Channel Flow** | $\mathbf{k_c}$ |
| 1. Meandering stream with some pools (n = 0.040) | 20 |
| 2. Rock-lined stream (n = 0.035) | 23 |
| 3. Grass-lined stream (n = 0.030) | 27 |

**"n" AND "k" Values for Travel Time Calculations**

***Figure 3-3.4***

- A culvert or bridge can act as a reservoir outlet if there is significant storage behind it. A hydrograph should be developed to this point and the level pool routing technique described in Section 3-3.4 should be used to determine the outflow rating curve through the culvert or bridge.

***Example***

Calculate the travel times for each reach and the time of concentration for a drainage basin near Tacoma.

Given: P2 = 2.0 inches.

Segment 1: L = 100 ft. Forest with light brush (sheet flow)
$s_o$ = 0.03 ft/ft
$n_s$ = 0.40

Segment 2: L = 300 ft. Pasture (shallow concentrated flow)
$s_o$ = 0.04 ft/ft
ks = 11

Segment 3: L = 300 ft. Grassed waterway (intermittent channel)
$s_o$ = 0.05 ft/ft
$k_c$ = 17

Segment 4: L = 500 ft. Grass-lined stream (continuous)
$s_o$ = 0.02 ft/ft
$k_c$ = 27

Segment 1: Sheet flow $T_t = (0.42\ (n_sL)^{0.8})/((P2)^{0.527}(so)^{0.4})$
$T_1 = ((0.42)((0.40)(100))^{0.8})/((2.0)^{0.527}(.03)^{0.4}) = 23$ min

Segment 2: Shallow concentrated flow $V = k_s\ s_o\ ^{0.5}$
$V_2 = (11)(0.04)^{0.5} = 2.2$ ft/s
$T_2 = L/60V = 300/(60)(2.2) = 2$ min

Segment 3: Intermittent channel flow
$V_3 = (17)(0.05)^{0.5} = 3.8$ ft/s
$T_3 = 300/(60)(3.8) = 1$ min

Segment 4: Continuous stream
$V_4 = (27)(0.02)0.5 = 3.8$ ft/s
$T_4 = 500/(60)(3.8) = 2$ min
$T_c = T_1 + T_2 + T_3 + T_4$
$T_c = 23 + 2 + 1 + 2 = 28$ minutes

If the computer program WaterWorks is used, then the travel time and time of concentration values do not have to be calculated by hand. When entering the parameters that describe a drainage basin, a time of concentration calculator can be called up. This calculator can handle up to five different land segments. The input for the calculator is the same input that is required for the above example. Once finished inputting the information, the time of concentration calculator will compute the travel times, sum them up, and insert the time of concentration into the correct place in the basin parameters menu.

### 3-3.3 Hydrograph Synthesis

The SBUH method applies the CNs selected to SCS equations to compute soil absorption and precipitation excess from the rainfall hyetograph. Each time step of this process generates one block of an instantaneous hydrograph with the same duration. The instantaneous hydrograph is then routed through an imaginary reservoir with a time delay equal to the basin time of concentration. The end product is the runoff hydrograph for that land segment.

The abstraction of runoff, S, is computed from the CN as follows:

$$S = (1000/CN) - 10$$

Using the abstraction value and precipitation for the given time step, the runoff depth, D, per unit area is calculated as follows:

$$D_{(t)} = (p_{(t)} - .2(S)^2)/(p_{(t)} + .8(S))$$

where:

$p_{(t)}$ = precipitation for the time increment (in)

The total runoff, $R_{(t)}$, for the time increment is computed as follows:

$$R_{(t)} = D_{(t)} - D_{(t-1)}$$

The instantaneous hydrograph, $I_{(t)}$, in cfs, at each time step, dt, is computed as follows:

$$I_{(t)} = 60.5\ R_{(t)}\ A/dt$$

where:

A = area (acres)
dt = time interval (min)

**Note:** A time interval of 10 minutes is used for design storms of 24-hour duration.

The runoff hydrograph, Q(t), is then obtained by routing the instantaneous hydrograph I(t), through an imaginary reservoir with a time delay equal to the time of concentration of the drainage basin. The following equation estimates the routed flow, Q(t):

$$Q_{(t+1)} = Q_{(t)} + w[I_{(t)} + I_{(t+1)} - 2Q_{(t)}]$$

where:

$w = dt/(2T_c + dt)$
$T_c$ = Time of concentration for the drainage basin

***Example***

The design process for the SBUH method is fairly easy to program into a spreadsheet or to write a stand-alone computer application. While the calculations can be performed using a standard hand-held calculator, it is not advisable due to the large amount of calculations that must be executed. This example is done with the use of the computer program WaterWorks.

Calculate two runoff hydrographs for a project site near Tacoma. The first hydrograph is for the 100-year design storm with the site in its existing conditions. The second hydrograph uses the same design storm for the site after an interchange has been constructed.

Given:

Existing Site — 10 acres forested

Future Site — 3.9 acres impervious, 6.1 acres grassed

100 year rainfall — 4.0 inches for 24 hour duration

Soils — 80% Alderwood, 20% Melbourne

The first thing that must be done for this example is to start the WaterWorks program. Before any data can be input, a data set for the project must be created. Do this by selecting the File option from Main Menu, then select New Data from the File Menu. A prompt appears for a name for the project. For this example, use the name “TACOMA” as the name of the data set.

Now that a data set has been created, the project information must be input. Since the project is west of the Cascade Range, Type 1A storm must be used. The isopluvial maps indicated that the 100-year, 24-hour rainfall totals 4.0 inches for the project site.

The two soil types present on the project site are similar enough that they can be combined and analyzed as one. Alderwood is Type C soil with a CN of 73, Melbourne is Type B soil with a CN of 60. The weighted average is:

$$CN = (.80)(73) + (.20)(60) = 70$$

To enter the data, select Basin from the Main Menu, then select Basin from the Basin Menu. This will bring up a blank input screen. Each basin or subbasin will need a unique set of data input into this area. It is advisable to also create a unique basin definition for the different design storms of each basin in the project. By doing this, the designer can quickly analyze different scenarios without having to change the information in the Basin Menu.

Enter the data for the basin into the input screen for the existing site conditions.

| | |
|---|---|
| Basin ID ............................ : | A1 |
| Description ......................... : | 100-year storm existing conditions |
| Area (acres) ........................ : | 10 |
| Rain Precip (in) .................. : | 4.0 |
| Time Interval (min) ........... : | 10 |
| Time Of Conc (min) .......... : | 28 |
| Rainfall Selection .............. : | 1 |
| Abstrac Coeff ..................... : | 0.20 |
| Base Flow .......................... : | 0.00 |
| Storm Dur (hrs) .................. : | 24 |
| Pervious Area ..................... : | 10.00 |
| Pervious CN ....................... : | 70 |
| Impervious Area ................ : | 0.00 |
| Impervious CN ................... : | 98.00 |

A short explanation of the input parameters follow:

**Basin Data** — This is the way that the program keeps track of the basin, any alpha numeric combination will work here. This name will be the way that the basin is accessed by other parts of WaterWorks.

**Description** — This is a description that is provided only for the convenience of the designer to serve as a reminder of the data that was input. Any short text string will work.

**Area** — This is the total area of the basin in acres.

**Rain Precip** — This is the total depth of the design storm hyetograph. The value used for this example was obtained in Example 3-3.1.1.

**Time Interval** — This is the interval, in minutes, at which the SBUH calculations are made. Use 10 for 24-hour analysis.

**Time of Conc** — This is the time of concentration used. The value for this example is the same as that in Example 3-3.2.2. The easiest way to establish this value is to highlight this menu selection then press the [F4] key. A time of concentration calculator will appear which uses the same input as the equations described earlier in this chapter. When finished with the calculator, press the [F10] key and the calculated value will be automatically input into the Time of Conc field.

**Rainfall Selection** — This is the dimensionless hyetograph that is selected for the SBUH method. If the analysis is for a project site in eastern Washington, then the Type II storm should be selected.

**Abstract Coeff** — Should remain at 0.20.

**Base Flow** — This provides the ability to add a constant base flow to the generated hydrograph.

**Storm Duration** — This is the total amount of time that the storm event will occur over.

**Pervious Area** — This is the total pervious area of the basin in acres.

**Pervious CN** — This is the corresponding SCS curve number for the pervious land segment.

**Impervious Area** — This is the total impervious area of the basin in acres. This value is automatically updated as the Pervious Area value is changed.

**Impervious CN** — This is the corresponding SCS curve number for the impervious land segment, typically this should be 98.

Once this data is input, the [F6] key can be selected and some quick calculations will be performed. Near the bottom of the screen three values for the basin will be displayed. These are:

Peak Hydrograph Time = 8.17 hrs

Peak Hydrograph Flow = 1.61 cfs

Total Hydrograph Vol = 1.10 Ac-ft.

At this point, the program has not generated a hydrograph that can be used, it has only generated a summary that can be used to quickly check the basin. To save the basin information that was just input, press the [PgDn] key. Since basin A1 has been saved, the second basin for this example can now be input by overwriting the portions of basin A1 that are different.

Enter the data for the basin into the input screen for the project site conditions.

| | | |
|---|---|---|
| Basin ID ............................ | : | A2 |
| Description ......................... | : | 100 year storm project conditions |
| Area (acres) ........................ | : | 10 |
| Rain Precip (in) .................. | : | 4.0 |
| Time Interval ..................... | : | 10 |
| Time Of Conc (min) .......... | : | 12 |
| Rainfall Selection .............. | : | 1 |
| Abstrac Coeff ..................... | : | 0.20 |
| Base Flow .......................... | : | 0.00 |
| Storm Dur (hrs) .................. | : | 24 |
| Pervious Area ..................... | : | 6.10 |
| Pervious CN ....................... | : | 77 |
| Impervious Area ................ | : | 3.90 |
| Impervious CN .................. | : | 98.00 |

Notice that the Time of Conc has changed. This is a result of sheet flow on a smoother surface than in the existing conditions. Also, the Pervious CN has increased due to the change in land cover. Once again, select the [F6] key to display the quick results:

Peak Hydrograph Time = 8.00 hrs

Peak Hydrograph Flow = 5.14 cfs

Total Hydrograph Vol = 2.13 Ac-ft.

Now that both basins have been input, they can be used to create a working hydrograph. From the Main Menu select Hydrograph, then from the Hydrograph Menu select Add/Subt Hyd. This will bring up another small menu. Select the Move Basins option from this menu. This option will present a table that permits up to 10 defined basin hydrographs to be moved to hydrograph storage registers. WaterWorks has the ability to create as many defined basins as the designer has a need for. However, only 20 hydrographs can be made available for use at any one time. As a result, during a large project the designer may have to repeat this “move” step several times to completely analyze a basin. Fill in the opened input table so basin A1 is moved to hydrograph register 1 and basin A2 is moved to hydrograph register 2. To actuate the move command, press the [F6] key, then press [F10] until the prompt is at the Main Menu.

To view the hydrographs, select Hydrograph from the Main Menu, then select List hydro from the Hydrograph Menu. There are two options for listing a hydrograph. The first is a graphical display, the other is a listing of the peak values. Since the peak values are already known, the Graphic option should be selected. A table will pop up with the numbers 1 through 20 on it. These numbers represent the

20 hydrograph registers that are available within WaterWorks. Highlight the 1 and 2 by moving the cursor underneath them with the arrow keys and pushing the [Enter] key. When both are highlighted, press the [F10] key and both hydrographs will be graphical displayed.

### *3-3.4 Hydrograph Routing*

This section presents the methodology for routing a hydrograph through a stormwater facility using hydrograph analysis. Hydrograph routing is done the same way regardless of the method used to generate the hydrograph. Therefore, this part of the analysis is no longer unique to the SBUH method. The level pool routing technique presented here is one of the simplest and most commonly used hydrograph routing methods. It is based on the continuity equation:

Inflow — Outflow = Change in Storage

$$((I_1 + I_2)/2) - ((O_1 + O_2)/2) = S_2 - S_1$$

where:

I = Inflow at time 1 and time 2
O = Outflow at time 1 and time 2
S = Storage at time 1 and time 2

The time interval for the routing analysis must be consistent with the time interval used in developing the inflow hydrograph. The time interval used for a 24-hour storm is 10 minutes. The variables can be rearranged to obtain the following equation:

$$I_1 + I_2 + 2S_1 - O_1 = O_2 + 2S_2$$

If the time interval is in minutes, the units of storage (S) are now [cf/min] which can be converted to cfs by multiplying by 1 min/60 sec.

The terms on the left-hand side of the equation are known from the inflow hydrograph and from the storage and outflow values of the previous time step. The unknowns O and S can be solved interactively from the given stage-storage and stage-discharge curves. As with the synthesis of a hydrograph, the computations are fairly simple but very voluminous. The best way to route a hydrograph through a stormwater facility is to use a computer program. WaterWorks has many features that make hydrograph routing an easy process.

***Example***

Using the hydrographs developed in the previous example, develop a detention pond and route the project runoff hydrograph through it.

Given: The pond is allowed to release at a rate after the project is constructed that is equal to the rate of runoff prior to the construction of the project. A dry pond should be used as the storage structure with a maximum depth of 5 feet and side slopes of 4:1.

A pond must exist before a hydrograph can be routed through it. The pond must also have an outlet structure which will release the flow from the pond at the allowed release rate. The amount of storage required is a function of the shape of the inflow hydrograph and the allowed release rate. A good first estimate of required volume is the difference between the existing conditions hydrograph's total volume and the project hydrograph's total volume. The outlet structure is based on the maximum allowed release rate. In this case, the peak flow of the existing conditions hydrograph.

Estimated Pond Volume = $V_{A2}$ — $V_{A1}$ = 2.13 — 1.10 = 1.03 Ac-ft

Maximum Release Rate = $Q_{pA1}$ = 1.61 cfs

Start WaterWorks from the Main Menu select File, then from the File Menu select Open Data. A box will be displayed that will show all of the previously created data sets. Highlight the one that is titled TACOMA and press the [Enter] key. These steps can be skipped if WaterWorks was not shut down since following through the last example on hydrograph synthesis.

From the Main Menu, select Storage. This will display the Storage Menu. There are three predefined storage structures to select from and two options for creating customized structures if none of the predefined ones will work for the given design situation. For this example, select Trapezoidal basin. A trapezoidal basin as used in this context is an open pond that is rectangular in plan view and trapezoidal in cross section. To obtain the estimated 1.03 Ac-ft of storage required, fill in the Trapezoidal basin item as follows:

Storage Structure ID .......... : S1
Name .................................. : Detention Pond
Length ................................ : 85
Side Slope1 ........................ : 4
Side Slope2 ........................ : 4
Width.................................. : 65
Side Slope3 ........................ : 4
Side Slope4 ........................ : 4
Infiltration Rate.................. : 0
Starting Elev ...................... : 325.0
Max Elev ............................ : 330.0
Stg-Sto Increm ................... : 0.1

**Storage Structure** — This is the name the computer will use for this pond. Any alpha numeric combination will work.

**Name** — This is a description that is provided only for the convenience of the designer. It serves as a reminder about which pond this is.

**Length** — A trapezoidal basin is rectangular in plan view and trapezoidal in cross. This is the distance along one dimension of the bottom of the pond as observed in plan view.

**Side Slope1** — This is the slope of one side of the pond along the length dimension. Each of the four sides of a trapezoidal basin can be input with a unique slope. The input is in the form of a ratio of x:1, where only the x value needs to be included.

**Side Slope2** — This is the slope of the other side of the pond along the length dimension.

**Width** — This is the distance along the other dimension of the bottom of the pond.

**Side Slope3** — This is the slope of one side of the pond along the width dimension.

**Side Slope4** — This is the slope of the other side of the pond along the width dimension.

**Infiltration Rate** — This is the infiltration rate of the soil in the bottom of the pond in minutes per inch. If this rate is known then it should be input even if the pond is not intended to be an infiltration pond. If no information about the infiltration rate of the soil is available then a zero should be input. It is usually a good design procedure to only input 50 percent of the measured infiltration rate. This will account for the reduction in infiltration that often occurs when fines accumulate along the bottom of the pond.

**Starting Elev** — This is the elevation where calculations start. Typically this is the bottom of the pond.

**Max Elev** — This is the elevation where calculations stop. Typically this is the top of the pond or overflow elevation.

**Stg-Sto Increm** — This is the increment in feet that the stage-storage calculations are done at. For ponds that are no deeper than six feet, use 0.1 for this value.

Near the bottom of the screen the volume of the pond is displayed. This should nearly match the estimated value.

The next step in designing a detention pond is to specify an outlet structure for the pond. From the Main Menu, select Discharge. This will display the Discharge Menu. There are seven predefined discharge structures to select from and an option for creating customized structure if none of the predifined structures match the situation under design. For this example, select Orifice design. WaterWorks will automatically design the correct orifice size given an input hydrograph, a peak flow to match, and a storage facility. To allow for an orifice design, only input the general characteristics of the discharge structure into the input screen.

Discharge Structure ID ...... : D1

Elev of Lowest Orifice ...... : 325.0

Outlet Elev ......................... : 325.0

Max El Above Outlet......... : 330.0

Stg-Disch Increm ............... : 0.1

**Discharge Structure ID** — This is the name the computer will use for this outlet. Any alpha numeric combination will work.

**Elev of Lowest Orifice** — This is the elevation of the lowest or first orifice.

**Outlet Elev** — This is the elevation of the outlet pipe or channel. The flow out through any orifice is calculated relative to this elevation.

**Max El Above Outlet** — This is the elevation where calculations stop. Typically this matches the top of the pond that the discharge structure serves.

**Stg-Disch Increm** — This is the increment in feet that the stage-discharge calculations are done at. For ponds that are no deeper than six feet, use 0.1 for this value.

Now that a detention facility has been defined, a hydrograph can be routed through it. This is done using the level pool routing technique. From the Main Menu, select Level pool. This will bring up a menu of different routing options. Select Input table to define the hydrograph to be routed, the storage structure to route the hydrograph through, and the discharge structure to use for the outlet. An input table will appear that has several lines on it. This provides the ability to do several level pool routing processes at one time. For this example only, the first line needs to be filled in as follows:

Description ......................... : 100-year storm
Pre Hyd # ........................... : 1
Inflo Hyd # ......................... : 2
Stg Stor ID ......................... : S1
Stg Dis ID .......................... : D1
Outflow Hyd # ................... : 10

A short explanation of the input parameters follow:

**Description** — This is a description provided for the use of the designer. The computer program does not use this parameter for any calculations.

**Pre Hyd #** — This item is not required for level pool routing unless an orifice design is being performed. During an orifice design, WaterWorks will use the peak flow value of the indicated hydrograph as the maximum allowed release rate of the storage structure. If the designer prefers to match the outflow of a pond to a value other than the peak of one of the hydrographs, a real number can be input. To input a real number, a decimal point and a tenth value must be included so WaterWorks can distinguish it from a hydrograph number.

**Inflow Hyd #** — This is the number of the hydrograph to route through the storage structure.

**Stg Stor ID** — This is the storage structure through which the hydrograph will be routed.

**Stg Dis ID** — This is the outlet structure that will control flow from the storage structure. In the case of this example, WaterWorks will calculate the size of the orifice to use as an outlet.

**Outflow Hyd #** — This is the number of the hydrograph that was created during the level pool routing process. Once created, it can be viewed through the use of the View hydro.

Once all of the parameters have been input, press the [F10] key to return to the Level pool Menu. Since this example is designing an orifice, the analysis selection must be Basin Design. If an orifice size had been given for the outlet structure or if a weir had been designed as the outlet structure, then the selection from this menu would have been Compute table.

When the Basin Design option is selected, a box will appear on the screen. WaterWorks will step through a number of iterations until it selects an orifice size that allows a peak outflow no greater than the flow rate indicated in the Pre Hyd # input parameter. Once this process is finished, press the [F10] key to return to the Level pool Menu. Select the Display table option to display the results of the analysis.

Orifice Size = 6.12 in (155 mm)

Pond Depth = 2.51 ft (770 mm)

### 3-3.5 Multiple Design Storm Routing

The example presented in the previous section is an accurate but simplified version of the hydrograph routing that must be done for an actual detention facility to meet the requirements of this manual. To meet Minimum Requirement 5 in Chapter 2, the outlet of the detention facility must be sized based on three different storms. For the existing conditions of the site, runoff must be calculated for the 2-year, 10-year, and 100-year return frequency storms. The runoff from the site after the project is completed can be no greater during the 2-year storm than 50 percent of the 2-year existing conditions runoff, no greater during the 10-year storm than the 10-year existing conditions runoff, and no greater during the 100-year storm than the 100-year existing conditions runoff.

As a result of this multiple storm requirement, a different approach to designing an outlet structure is used. Instead of designing a single outlet for the detention facility, multiple outlets are designed to allow for increased outflow during larger storm events. The easiest approach to solving this problem (especially if WaterWorks is used for the design) is to size three different orifices. The first orifice would be placed near the bottom of the pond and would be sized by routing a 2-year storm through the detention facility. The second orifice would be placed just above the maximum surface elevation that occurs during the 2-year storm and would be sized by routing a 10-year storm through the detention facility. The third orifice would be placed just above the maximum surface elevation that occurs during the 10-year storm and would be sized by routing the 100-year storm through the detention facility.

Another method of designing a detention facility outlet is to use a combination of an orifice and a weir, or a combination of an orifice and a standpipe. The orifice would be designed by routing a 2-year storm through the detention facility. The weir or standpipe would be designed by routing the 100-year storm through the detention facility. The 10-year storm would then have to be routed through the facility to ensure that its release did not exceed the allowed rate.

The triple orifice configuration is slightly easier to design than the combination of orifice and weir, mainly because WaterWorks will actually design an orifice size but will not design a weir size. The advantage of the orifice and weir combination is that often a smaller volume for the detention facility is needed. Also, a weir or standpipe is less prone to becoming plugged due to debris.

Whenever designing an orifice as an outlet structure, the diameter selected should never be less than 1 inch. This will create problems on small sites since they will have allowed release rates that will dictate the use of a smaller orifice. However, an orifice smaller than 1 inch becomes plugged too easily to be considered practical from a maintenance standpoint. If the site has a very low allowed release rate, then a weir outlet or infiltration should be used.

***Example***

Design a pond for a site in Tacoma that follows the design criteria of Minimum Requirement 5. Use a triple orifice outlet structure as the control.

Given:

Same site as Example 3-3.4.1.

10-year rainfall = 3.0 in

2-year rainfall = 2.0 in

To begin the analysis, start WaterWorks and open the data set named TACOMA. Four additional hydrographs have to be generated for this design, two more hydrographs for the existing conditions, and two more hydrographs for the project conditions. The best way to do this is to create four additional basins. Use the existing basins as templates and write over the input parameters that change. The only changes will be the Basin ID, the Description, and the Rain Precip. Enter the data as follows:

Basin ID ............................ : B1 (copy other data from Basin A1)
Description ......................... : 10-year storm existing conditions
Rain Precip ......................... : 3.0

Basin ID ............................ : C1 (copy other data from Basin A1)
Description ......................... : 2-year storm existing conditions
Rain Precip ......................... : 2.0

Basin ID ............................ : B2 (copy other data from Basin A2)
Description ......................... : 10-year storm project conditions
Rain Precip ......................... : 3.0

Basin ID ............................ : C2 (copy other data from Basin A2)
Description ......................... : 2-year storm project conditions
Rain Precip ......................... : 2.0

Press the [F6] for each of the existing conditions basins and record the peak runoff that is generated.

A1 = 1.61 cfs

B1 = 0.59 cfs

C1 = 0.20 cfs

A1 and B1 will be the allowed release rate for the project for the 100-year and 10-year storms respectively. Fifty percent of C1 will be the allowed release rate from the project site during the 2-year storm.

Allowed Release Rate:

100-year storm = 1.61 cfs

10-year storm = 0.59 cfs

2-year storm = 0.10 cfs

Using the Move Basin command in the Hydrograph Menu, move all of the basins for the project conditions into a hydrograph register.

A2 — Hydrograph 1

B2 — Hydrograph 2

C2 — Hydrograph 3

Since a detention pond has already been defined (Stg Stor ID = S1) and since the new outlet design will write over the existing outlet structure (Stg Dis ID = D1), there is no need to define another detention facility. Select the Input table command from the Level pool Menu. Fill in the input table as follows:

**Level Pool Routing Instructions**

| **Description** | **Pre Hyd #** | **Inflo Hyd #** | **Stg Stor** | **Stg Dis** | **Outfl Hyd #** |
|---|---|---|---|---|---|
| [2-year design storm ] | 0.10 | 3 | S1 | D1 | 13 |
| [10-year design storm ] | 0.59 | 2 | S1 | D1 | 12 |
| [100-year design storm] | 1.61 | 1 | S1 | D1 | 11 |
| [ ] | [ ] | [ ] | [ ] | [ ] | [ ] |
| [ ] | [ ] | [ ] | [ ] | [ ] | [ ] |
| [ ] | [ ] | [ ] | [ ] | [ ] | [ ] |
| [ ] | [ ] | [ ] | [ ] | [ ] | [ ] |

Once this is input, press [F10] to return to the Level pool Menu. Then select the Basin Design option from the menu. WaterWorks will run through several iterations as it designs an orifice for each storm event. When this process is finished, press the [F10] key to return to the menu and select Display table to determine the maximum water surface elevation during the 100-year storm. This will show how deep to make the final pond. Return to the Main Menu and select Discharge. Then select View Dis-Struct. This will display the sizes of the orifices and the elevations at which to locate them. The final pond design should be as follows:

Length = 85 ft (25.9 m)

Width = 65 ft (19.8 m)

Max Depth = 4.43 ft (1.35 m)

1st Orifice: Size = 1.39 in (35 mm)
Elevation = 325.0 ft

2nd Orifice: Size = 4.84 in (123 mm)
Elevation = 328.7 ft

3rd Orifice: Size = 20.81 in (529 mm)
Elevation = 329.4 ft

This example has essentially designed a Dry Pond (BMP RD.11). The final step in designing this type of BMP is to apply the appropriate factor of safety. Since 39 percent of the project site is impervious, the factor of safety is 1.28 (see Figure 2-6.2). The pond volume must be increased by 28 percent without increasing the average depth. Also it should be noted that the top orifice, with a diameter of 20.8 inches, would typically be impractical to construct. The orifice should be reduced to a more practical size and the hydrographs should be routed through the pond again.

## 3-3.6 Hydrograph Summation

One of the key advantages of hydrograph analysis is the ability to accurately describe the cumulative effect of runoff from several basins or subbasins having different runoff characteristics and travel times. This cumulative effect is best characterized by a single hydrograph which is obtained by summing the individual hydrographs from tributary basins at a particular discharge point of interest.

The general procedure for performing a hydrograph summation begins with selecting a discharge point of interest where it is important to know the effects of the runoff generated on the project site. Next, route each individual hydrograph through conveyance system that carries it to the point of interest. The final step is to sum the flow values for each hydrograph for all of the time intervals. This will yield a single discharge hydrograph.

***Example***

Determine the effects of combining the flow from the detention pond designed in the example of Section 3-3.5 with another basin located adjacent to the project site. The event of concern is the 100-year storm.

Given:

Basin ID ............................ : A3
Description ......................... : 100-year storm off site
Area (acres) ........................ : 15
Rain Precip (in) .................. : 4.0
Time Interval ..................... : 10

Time Of Conc (min) .......... : 36
Rainfall Selection .............. : 1
Abstrac Coeff ..................... : 0.20
Base Flow .......................... : 0.00
Storm Dur (hrs) .................. : 24
Pervious Area ..................... : 15.0
Pervious CN ....................... : 65
Impervious Area ................ : 0.0
Impervious CN ................... : 98.00

Begin by starting WaterWorks and opening the TACOMA data set. Enter the new basin information for Basin A3. The easiest way to do this would be to write over Basin A1 or A2 since much of the information is the same. Once the basin is defined, it needs to be moved to a hydrograph storage register. From the Main Menu, select Hydrograph, then Add/Subt Hyd, then Move. From the Move input box, move Basin A3 into hydrograph register 5. Press [F10] to return to the Add/Subt Hyd Menu.

The other hydrograph that is to be combined with the newly generated one is the output from the detention pond during the 100-year storm. The last example stored this hydrograph in register 11. Since the two basins are located next to each other, there is no need to route either hydrograph through a conveyance system. From the Add/Subt Hyd Menu, select the Add Hydrograph option. Fill in the input table such that hydrograph 5 is added to hydrograph 11 to equal hydrograph 6. Press [F10] twice, then view the results using the List hydro option on the Hydrograph Menu.

Hydrograph 6

Peak Flow = 2.71 cfs (0.08 cms)

Time of Peak Flow = 10.33 hours

Total Volume = 2.89 Ac-ft (0.36 ha-m)

### *3-3.7 Widening Projects*

The example that has progressed through most of this chapter deals with an idealistic situation where the roadway being constructed is completely new. This is the simplest scenario for stormwater runoff design because all of the runoff from the newly constructed road is routed through the treatment facility.

A more common project would involve the improvement of a roadway section such as adding lanes or widening shoulders. Currently, WSDOT projects are required to treat the stormwater runoff from any new impervious surfaces that are created by the project. If the runoff from existing impervious surfaces is treated, then the project is considered to be retrofitted for stormwater treatment. At some time in the future, all WSDOT roadways will be retrofitted for stormwater treatment to the extent practicable.

The designer should always consider the possibility of retrofitting a project site. There are times when a retrofit can be accomplished for very little additional cost. There are also times when even though the cost of retrofitting is high, doing so will create a large benefit to a natural resource. For all projects that involve stormwater treatment, the designer must determine the cost of retrofitting the entire roadway section and record the estimated cost in the Hydraulics Report. Information on the benefits that will be gained by retrofitting a specific project can be obtained from the regional Environmental Section, the Department of Fish and Wildlife, the Department of Ecology, and many local agencies.

The remainder of this section deals with the situation where only the runoff from the newly created impervious surface will be treated. The biggest problem that occurs with this scenario is that runoff from the new roadway will mix with runoff from the existing roadway.

There is no possible way to separate out the runoff from the new roadway area once it has mixed with runoff from other areas. However, it is possible to treat an amount of runoff from both sources that is equal to the volume of runoff from the new impervious area. For water quantity treatment, this can be accomplished by sizing the outlet device to have a maximum flow out equal to the sum of the peak flow of the existing area runoff and the allowed peak flow for the new area runoff.

While stormwater detention, or water quantity treatment for a widening project can be handled by properly adjusting the release rate of the pond, water quality facilities must use a different approach. Water quality facilities rely on a set volume or a set area for treatment. If extra flow is introduced, the pollutant removal of the facility will not be as high as expected. For a water quality facility to operate correctly, it can only accept flow from the amount of area that it was designed for. This can be accomplished in a couple different ways.

One method would be to create a diversion structure and separate conveyance system which allows an amount of flow equal to that from the existing area to bypass the stormwater facility. A better method would be to treat some of the existing area runoff and only a portion of the new runoff. For example, if there was a ½-mile section of roadway that was going to be widened from two lanes to four lanes, a water quality facility could be designed in either of two ways. The design could follow the first method discussed and separate out the flow from the two new lanes for the entire ½-mile section and route that flows through a facility. The design could otherwise follow the second method and route the flow from all four lanes for the first ¼-mile section of the project through a facility. Either way, a total of one-lane mile is being treated, which is equal to the amount being constructed. The second method is preferred because it would be less expensive to construct and would be easier to incorporate into a program of retrofitting existing highway sections with stormwater facilities. It is important to always consider the future possibility of retrofitting and what can be done to make that process easier.

***Example***

A section of highway near the city of Vancouver is scheduled to be improved with an additional lane in each direction. The existing configuration consists of two 12-foot lanes with a 6-foot shoulder on one side and an 8-foot shoulder on the other in each direction. The northbound and southbound lanes are separated by an existing 50-foot grassed median. All of the pavement is sloped towards the median which is “V” shaped and acts as the conveyance system. There are ditches outside of the lanes which prevent off-site drainage from entering the project area. The proposed project will add one 12-foot lane in each direction, while maintaining the current shoulder widths. Both lanes will be added inside of the existing lanes.

The soil in the project vicinity has an SCS identification as Elochoman. The median area is grassed with no appreciable amount of brush. Using this information, design a detention pond for this 2,500-foot section of roadway that will meet the required release rates of Minimum Requirement 5.

The rainfall for the Vancouver area is obtained from Figure 3-3.1. The rainfall amounts are 2.3 inches for the 2-year recurrence, 3.0 inches for the 10-year recurrence, and 4.3 inches for the 100-year recurrence.

Figure 3-3.3 shows that Elochoman soil is SCS Type B soil. A curve number can be obtained from Figure 3-3.2 for the pervious land segments. 98 is always used as the curve number for the paved areas. Given the described conditions of the median, a curve number of 58 should be used.

Calculating the area contributing to the pond being designed indicates that there is a total of 7.23 acres. The existing roadway configuration has 4.36 acres of impervious and 2.87 acres of pervious land contributing flow. After the project is completed, there will be 5.74 acres of impervious and 1.49 acres of pervious land contributing flow.

To best analyze this basin for the tristorm requirement, the designer should define 6 basins within WaterWorks. The 2-year storm, the 10-year storm, and the 100-year storm will each use two basins, one to model the existing conditions and one to model the project conditions.

The basins would be entered into WaterWorks as follows:

Basin ID ............................ : A1
Description ......................... : 2YR EXISTING CONDITIONS
Area (acres) ........................ : 7.23
Rain Precip (in) .................. : 2.3
Time Interval ..................... : 10
Time Of Conc (min) .......... : 24.21
Rainfall Selection .............. : 1
Abstrac Coeff ..................... : 0.20
Base Flow .......................... : 0.00
Storm Dur (hrs) .................. : 24
Pervious Area ..................... : 2.87
Pervious CN ....................... : 58
Impervious Area ................ : 4.36
Impervious CN ................... : 98

(Repeated Information is not Shown for Remaining Basins)

| | | |
|---|---|---|
| Basin ID ............................. | : | A2 |
| Description ......................... | : | 2YR PROJECT CONDITIONS |
| Time Of Conc (min) .......... | : | 20.55 |
| Pervious Area ..................... | : | 1.49 |
| Impervious Area ................ | : | 5.74 |
| | | |
| Basin ID. ............................ | : | B1 |
| Description ......................... | : | 10YR EXISTING CONDITIONS |
| Time Of Conc (min) .......... | : | 24.21 |
| Pervious Area ..................... | : | 2.87 |
| Impervious Area ................ | : | 4.36 |
| | | |
| Basin ID ............................. | : | B2 |
| Description ......................... | : | 10YR PROJECT CONDITIONS |
| Time Of Conc (min) .......... | : | 20.55 |
| Pervious Area ..................... | : | 1.49 |
| Impervious Area ................ | : | 5.74 |
| | | |
| Basin ID ............................. | : | C1 |
| Description ......................... | : | 100YR EXISTING CONDITIONS |
| Time Of Conc (min) .......... | : | 24.21 |
| Pervious Area ..................... | : | 2.87 |
| Impervious Area ................ | : | 4.36 |
| Basin ID ............................. | : | C2 |
| | | |
| Description ......................... | : | 100YR PROJECT CONDITIONS |
| Time Of Conc (min) .......... | : | 20.55 |
| Pervious Area ..................... | : | 1.49 |
| Impervious Area ................ | : | 5.74 |

Press the [F6] key each time the information is entered for a basin to display the peak rate of flow that will occur from the basin. The peak flow rates are mainly a concern for the existing conditions. The results will indicate 1.66 cfs for basin A1, 2.20 cfs for basin B1, and 3.28 cfs for basin C1. From these figures, the allowed release rates can be obtained. They will be 0.83 cfs for the 2-year storm (50 percent of basin A1), 2.20 cfs for the 10-year storm, and 3.28 cfs for the 100-year storm.

Hydrographs from the three project conditions basins will have to be routed through the pond being designed. Before any hydrographs can be routed, they have to be created. From the Main Menu select Hydrograph; from the Hydrograph

Menu select Add/Subt Hyd; and from the Add/Subt Menu select Move Basins. Move basin A2 to hydrograph 1; basin B2 to hydrograph 2; and basin C2 to hydrograph 3.

The next step in the design is to define a pond. The best first estimate for a pond volume is the difference between the volume of the existing conditions hydrograph for the 100-year storm and that of the project conditions hydrograph. When the [F6] key was used to display the peak runoff rates while in the basin, part of the program the volume of the storm is also displayed. For this example, the difference between the two 100-year hydrographs is approximately 0.4 acre-foot.

To define the pond, select the Trapezoidal Basin option from the Storage Menu. There are several different layouts that can be used for any pond. Typically, the site will dictate the shape of the pond; however, no one configuration is realistically any better than any other as long as it fits the site. Fill in the information as follows:

| | |
|---|---|
| Storage Structure ID .......... : | S1 |
| Name .................................. : | Detention Pond |
| Length ................................ : | 70 |
| Side Slope1 ........................ : | 4 |
| Side Slope2 ........................ : | 4 |
| Width .................................. : | 10 |
| Side Slope3 ........................ : | 4 |
| Side Slope4 ........................ : | 4 |
| Infiltration Rate .................. : | 0 |
| Starting Elev ...................... : | 200.0 |
| Max Elev ............................ : | 205.0 |
| Stg-Sto Increm ................... : | 0.1 |

The final thing that must be defined before routing the hydrographs through the pond is the discharge structure. Instead of using a three-orifice configuration, as was done in the last example, this pond will use one orifice in the side of a standpipe. The orifice will control the 2-year storm. The standpipe will control the 10-year storm and the 100-year storm. This will be done by designing the orifice alone using the 2-year storm hydrograph then designing the standpipe overflow in conjunction with the orifice using the 100-year storm hydrograph. The system is then checked to ensure that the 10-year storm release rate is less than the maximum allowed.

The design of the orifice is simply done by using the Basin Design option of the Level Pool Menu, the same way it was done in the previous example. The design of the standpipe is best done with a trial and error analysis of the design storm. A Riser (standpipe) is defined such that its top elevation is at least 0.10 feet higher than the 2-year water surface elevation. The 100-year storm is then routed through the pond and the maximum rate of discharge from the pond is checked against the allowed rate. If the actual rate is larger than the allowed rate, then the riser diameter can be decreased or the top elevation can be increased. This process is repeated until the flows match.

To accomplish this, three outlet structures have to be defined.

(Orifice Design)

Discharge Structure ID ...... : D1

Name ................................. : Orifice Outlet

Lowest Orifice Dia (in) ..... : 4.0

Elev of Lowest Orifice ...... : 200.0

Outlet Elev ......................... : 200.0

Max El above Outlet .......... : 205.0

Stg-Sto Increm ................... : 0.1

(Riser Inflow)

Discharge Structure ID..: D2

Description ......................... : Standpipe

Riser Diameter (inches) ..... : 10

Riser Elevation (feet) ......... : 204.0

Maximum Elevation (ft) .... : 205.0

(Combination)

Discharge Structure ID ...... : D3

Name ................................. : Combined

Structure 1 .......................... : D1

Structure 2 .......................... : D2

When these structures are defined, the hydrographs can be routed through the pond. To do this, use the Compute Table command from Level Pool Menu. The Input Table will define which storm is being analyzed. The three entries on the Input Table will be as follows:

Description ......................... : 2-year storm

Pre Hyd # ........................... :

Inflo Hyd # ......................... : 1

Stg Stor ID ......................... : S1

Stg Dis ID.......: .................. D3

Outflow Hyd # ................... : 18

Description ......................... : 10-year storm

Pre Hyd # ........................... :

Inflo Hyd # ......................... : 2

Stg Stor ID ......................... : S1

Stg Dis ID .......................... : D3

Outflow Hyd # ................... : 19

Description ......................... : 100-year storm

Pre Hyd # ........................... :

Inflo Hyd # ......................... : 3

Stg Stor ID ......................... : S1
Stg Dis ID .......................... : D3
Outflow Hyd # ................... : 20

Notice that the Pre Hyd# is left blank. This is only needed if an orifice is being designed with the Basin Design command. After the hydrographs are routed through pond, the display will show that release rates from the pond either match or are below the allowed release rates so the pond is sized correctly. In a real situation, it will take several trial runs to get the pond size correct and the outlet structures sized correctly to achieve the allowable release rates; however, these trial runs will only take a few minutes to perform once all of the other data has been entered.

The final step with this design would be to apply a factor of safety of 1.42 since 80 percent of the basin is impervious. This must be done without increasing the depth of the pond. The easiest way to do this is to determine the required volume and then change the length and width of the pond in the View Structure command of the Storage Menu. For this example, the highest elevation that the pond filled to during the 100-year storm was 204.44 feet. This gives the pond a total volume of 0.26 Ac-ft. Increasing that by 42 percent gives a required pond volume of 0.37 Ac-ft. By increasing the length of the pond to 90 feet and the width of the pond to15 feet, this volume is achieved.

As an additional note, the water quality treatment (Minimum Requirement 4) for this project would most likely be accomplished by using the median as a filter strip. Another very likely option would be a biofiltration swale immediately upstream of the detention pond.

## 3-4 Conveyance Systems

A conveyance system includes all portions of the surface water system, either natural or man-made, that transport stormwater runoff. The purpose of the conveyance system is to drain surface water from areas such that no damage will result from the flow. A properly designed system will maximize hydraulic efficiency while minimizing erosion and allowing for enhancement of water quality.

Open, vegetated channels are the preferred method of conveyance. The vegetation will help keep the channel from eroding and will provide some water quality treatment. Pipe systems are very efficient at transporting flow but offer no water quality benefits and are more difficult to maintain. As a result, pipe systems should only be used when open channel conveyance is not practical.

The WSDOT *Hydraulics Manual* discusses in detail the design methodologies for different types of conveyance systems. Since all of the material presented in the *Hydraulics Manual* concerning conveyance systems is still valid when used in conjunction with the material presented earlier in this chapter, the *Hydraulics Manual* should be used as the reference guide when designing a conveyance system.

The design of a conveyance system requires that the engineer knows the peak flow that will likely occur at the system. The Rational Method, as presented in the *Hydraulics Manual*, is the quickest way of determining the peak runoff flow for a project site. However, there are two instances when the SBUH method should be used instead of the Rational Method. The first instance is when the basin becomes too complex to accurately analyze with the Rational Method. This can occur from either the basin becoming too large (over 50 acres) or having several different land cover and soil types. The other instance is when the conveyance system is located downstream of a stormwater detention facility, the Rational Method does not have the ability to account for a reduction in flow resulting from detention.

It should be noted that for a basin with a short time of concentration (15 minutes or less) the Rational Method will yield a higher peak flow rate than the SBUH method. This is particularly true for basins in western Washington. The reason this occurs is that the Rational Method uses the highest possible peak intensity rainfall whether it occurs during a 3-hour thunderstorm or during a 7-day continuous rainfall event. The SBUH method on the other hand uses a specific duration, 24 hours. The longer duration storms do not have the extremely intense peaks that occur during storms of very short duration. As the time of concentration increases, the peak intensity that the Rational Method uses more closely matches the peak intensity of the SBUH hyetograph. This difference in flow values should not be of concern to the designer since basins with a time of concentration less than 15 minutes are fairly small and will typically be analyzed with the Rational Method.

To design a conveyance system using the SBUH method, develop a hydrograph for the area of interest as described previously in this chapter. Use the highest flow value from that hydrograph and proceed with the design steps given in the *Hydraulics Manual*.

Most computer programs that calculate hydrographs based on the SBUH method also offer the ability to design or analyze conveyance systems based on the generated hydrographs. WaterWorks offers the ability to analyze ditches, pipes, and gutters. A complex network of these different conveyance systems can also be analyzed. This feature can also be used to route a hydrograph through a long system causing the timing of the hydrograph flows to become shifted. The analysis of hydrograph shifting is important when analyzing the effects of a stormwater facility in a subbasin on a downstream location in the drainage basin.

***Example***

Design the outlet conveyance system for the detention pond designed in the Tacoma Example. The outlet will consist of 200 feet of CMP pipe leading from the pond and outletting to a channel. The channel will run 500 feet and empty into a nearby stream. The conveyance system should be designed for the 100-year storm.

Given:

Average Slope to Stream = .015 $^{ft}/_{ft}$

Manning’s n for CMP = .012

Manning’s n for Grassed Channel = .030

Discharge Stream’s 100-year Surface Elevation = 311.5

Begin by starting WaterWorks and opening the data set TACOMA. From the Main Menu, select Basin, and then select Pipe. This will bring up the input table for a pipe. Since a computer program offers the ability to quickly run “what if” scenarios, the best way to design a pipe or any other conveyance system structure is to assume a size and use the program to see if that size is correct. If not, use the results to select a different size. Using this philosophy, fill in the pipe input table as follows:

Reach Number .................. : R1
Pipe Diameter (in) ............ : 12
Pipe Length (ft)................. : 200
Pipe n Coefficient ............. : .012
Pipe Slope (ft/ft) ................ : .02
Upstream IE ....................... : 325
Downstream IE .................. : 321
Upstream Node ID............. : (leave blank)
Downstream Node ID ........ : (leave blank)
Contributing Basin............. : (leave blank)

Most of the input parameters are self explanatory, the ones that may not be are defined as follows:

**Upstream IE** — The invert elevation at the upstream end of the pipe. This is the elevation of the lowest inside part of the pipe.

**Downstream IE** — The invert elevation at the downstream end of the pipe. This is the elevation of the lowest inside part of the pipe. This value will be filled in automatically by WaterWorks if the slope and upstream elevation are filled in.

**Upstream Node ID** — This is the name of the node that the upstream end of the pipe is connected to. Nodes are only used when large pipe networks are being modeled and head losses through the nodes become significant. An example of a node would be a catch basin or a manhole.

**Downstream Node ID** — This is the name of the node that the downstream end of the pipe is connected to.

**Contributing Basin** — This is the name of the basin that will have its hydrograph routed through the pipe. Using this option will save the step of having to transfer the basin hydrograph to a hydrograph storage register. For this example, this option cannot be used since the hydrograph of interest has to be routed through a detention pond prior to being routed through the pipe.

When all of the information has been entered, press [F10] twice to return to the Main Menu. Select Hydrograph, then select Route hydro. This brings up the hydrograph routing input table, from the earlier example hydrograph 11 was created by routing the 100-year storm through a detention pond, route hydrograph 11 through reach R1 to analyze the pipe. Press [F6] to do the routing.

After the routing is complete, a table will be displayed which shows the design elements of the pipe. The table shows that a 12-inch CMP pipe at .02 ft/ft grade will work for the 100-year flow from the pond. If the pipe had been over capacity, WaterWorks would have alerted the designer.

Pipe Capacity = 27%

Pipe Velocity = 6.1 $^{ft}/_{sec}$

Next, the channel leading from the pipe to the stream must be designed. From the Main Menu, select Basin, and then select Ditch. This will bring up the input table for a channel reach. As with the pipe design, the best way to design a channel using WaterWorks is to estimate a channel size and see if it will work. If it does not, then change the size based on the answer from the first analysis. Input the information as follows:

Reach ID ............................ : R2
Ditch Length (ft) ................ : 500
Ditch Width (ft) ................. : 2
Side Slope1 (H:1V) ........... : 2
Side Slope2 (H:1V) ........... : 2
Ditch Slope ........................ : .02
Mannings n Coeff .............. : .040
Contrib Basin ..................... : (leave blank)
Downstream IE .................. : 311
Downstream WS ................ : 311.5
# of Trials........................... : 10

An explanation of the parameters that are different from previously used parameters follows:

**Ditch Width** — This is the width of the bottom of the channel.

**Side Slope(1 and 2)** — The side slope of the channel section, WaterWorks has the ability to analyze channels with different slopes on the right and left side.

**Ditch Slope** — The longitudinal slope of the channel.

**Downstream WS** — This is the controlling water surface elevation at the downstream end of the channel. For this example, it is the surface elevation of the stream that the channel discharges to.

**# of Trials** — WaterWorks conducts a step backwater analysis when analyzing a channel reach. This parameter is the maximum number of iterations that will occur.

Once the data is input, press [F10] twice. Select Hydrograph, then select Route hydro. This will bring up the same input table that was used to route a hydrograph through the pipe. Use hydrograph 11 again and route it through reach R2. When the analysis is complete, a table will appear that indicates the following:

Maximum Depth = 0.30 ft

Velocity in Channel = 2.01 ft/sec

This would indicate that a channel depth of approximately .5 foot (150 mm) should be used for this site.